Tunnelling and Underground Space Technology 24 (2009) 716–722

Contents lists available at ScienceDirect

Tunnelling and Underground Space Technology

journal homepage: www.elsevier.com/locate/tust

Technical note

A study on the effects of overlying soil strata on the stresses developing in a tunnel lining

M.A. Nunes, M.A. Meguid *

Department of Civil Engineering and Applied Mechanics, McGill University, 817 Sherbrooke Street West, Montreal, Quebec, Canada H3A 2K6

ARTICLE INFO

Article history:
Received 7 April 2009
Accepted 11 April 2009
Available online 9 May 2009

Keywords:
Tunnelling
Excavation
Overlying strata
Lining stresses

ABSTRACT

Shield tunnel lining in soft ground is usually designed based on empirical and analytical methods. These methods are convenient and easy to use; however, they are based on simplifying assumptions related to soil homogeneity around and above the excavated tunnel. The effects of overlying stiffer layers on the tunnelling induced settlement has been investigated by several researchers and was found to significantly influence ground movements. This study evaluates the effects of overlying stiff layers above a tunnel excavated in soft ground on the stresses developing in the tunnel lining. Laboratory investigation is conducted to study how the presence of these layers influence bending stresses in a model tunnel constructed in soft clay and overlain by a coarse sand layer located at different heights above the tunnel. Validation using the experimental results, elasto-plastic finite element analyses are then performed to explain the role of the relative stiffness between the overlying layer and the soft soil deposit hosting the tunnel. Depending on the thickness and location of the overlying stratum, the presence of a stiff layer above the tunnel can have a significant impact on the stresses developing in the tunnel lining.

1. Introduction

In engineering practice different methods are often used to calculate lining stresses, including empirical (e.g., Peck, 1969; Schmidt, 1974; Attwell, 1978; O'Reilly and New, 1982); analytical (e.g., Muir Wood, 1975; Einstein and Schwartz, 1979; Sagaseta, 1987; Verruijt and Booker, 1996; Bobet, 2001) and numerical analyses (e.g., Mair et al., 1981; Swoboda et al., 1989; Lee and Rowe, 1990; Addenbrooke and Potts, 2001). Among the above, empirical and analytical methods are considered to be simple and convenient tools to calculate the stresses in a tunnel lining, however, these methods are usually based on simplifying assumptions related to the soil homogeneity around the tunnel. Several physical models have been developed to study the ground response to tunnelling in homogenous soft ground including the trap door method (e.g., Terzaghi, 1936; Vardoulakis et al., 1981; Tanaka and Sakai, 1993), a pre-installed tube with vinyl facing (e.g., Chambon and Corte, 1994), a dissolvable polystyrene foam core (Sharma et al., 2001), or a miniature tunnel boring machine (Nomoto et al., 1999). These methods are described in more detail elsewhere (Meguid et al., 2008).

Many fine-grained soil deposits are naturally layered as a result of climatic cycles (Mitchell, 1976). The thickness of deposits formed during each cycle might vary from millimeters to several meters (Quigley et al., 1985). The presence of a coarse grained soil layer in homogenous clay deposit is known to affect ground response to tunnelling, particularly when the layer is located above the tunnel crown (see Fig. 1). The effect of soil layering on the ground response to tunnelling has been investigated by several researchers (e.g., Grant and Taylor, 2000; Hagiwara et al., 1999). Venkatachalam and Naik (1977) conducted a numerical investigation to study soil–tunnel interaction in stratified medium. The effects of position and orientation of the soil strata on the stresses developing in the lining was examined. Hagiwara et al. (1999) conducted a series of centrifugal tests by simulating the construction of a tunnel in clay overlain by a sand layer. Surface subsidence measurements were taken at the free surface of the overlying sand layer. It was found that the type and stiffness of the upper sand stratum has a significant effect on the movement of the underlying clay. Grant and Taylor (2000) performed a series of centrifuge model tests to study the stability of a tunnel in clayey soil overlain by a layer of coarse grained material. Results indicated that an upper stratum of loose sand material has no bearing on the tunnel stability, except that it acts as extra surcharge weight over the clay layer. Finally it was found that the presence of relatively dense sand layer enhanced the stability of the tunnel.

The focus of the above studies has been on tunnelling induced settlement and stability of the ground above the excavated tunnel. Little attention has been paid to the effects of the overlying strata on the stresses developing in the tunnel lining.

* Corresponding author. Tel.: +1 514 398 1537; fax: +1 514 398 7361.
E-mail addresses: miguel.nunes@mail.mcgill.ca (M.A. Nunes), mohamed.meguid@mcgill.ca (M.A. Meguid).

0886-7798/$ - see front matter © 2009 Elsevier Ltd. All rights reserved.
doi:10.1016/j.tust.2009.04.002

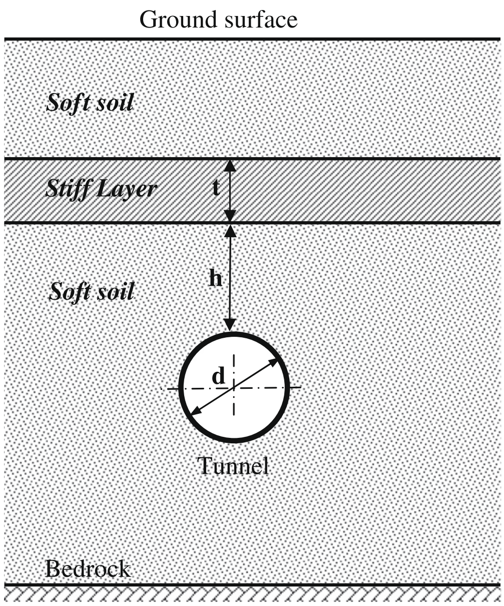

Fig. 1. A deep tunnel in soft soil overlain by a stiff layer.

The objective of this study is to investigate the changes in bending stresses in a tunnel lining installed in soft soil and overlain by a coarse grained sand layer. Experimental study is first conducted to measure the tunnelling induced stresses in a flexible lining system installed in clayey soil. A layer of sand was introduced at different heights above the tunnel crown. Two dimensional Elasto-plastic finite element analyses are then performed to study how the thickness, location and stiffness of this layer influence the bending moments in the lining. Emphasis in this study is placed on the short-term response of the lining following the tunnel construction.

2. Experimental investigation

A testing facility has been designed such that the entire model was contained in a rigid steel box (1.4 m in width; 1.2 m in height; and 0.3 m in thickness) with internally lubricated sides to minimize frictional restraint of soil movement. These dimensions were chosen to facilitate the two-dimensional tunnelling conditions. A side view of the rigid box is shown in Fig. 2. The tunnel location was selected such that overburden pressure applied over the tunnel is maximized and in the meantime the effect of the rigid boundary location on the measured lining stresses is minimized. This was achieved by placing the lateral boundaries at a distance of about four times the tunnel diameter (4*d*) from the tunnel circumference. The rigid base was located at a distance of 1.5*d* below the tunnel invert to represent the case where a strong layer (e.g., bedrock) is present. One side of the box was made of transparent Perspex, 12 mm in thickness, while the remaining four faces were made of steel, each 6 mm in thickness. Two caps were used to initially seal the tunnel opening. The caps were sawn in half to facilitate their removal during excavation. Circular rubber gaskets were also installed between the caps and the faces of the box to ensure tight connections.

An aluminum foil sheet 0.25 mm thick and 305 mm wide was rolled to form a thin-walled cylinder which was approximately 146 mm of outer diameter. A total of eight strain gauges were affixed to the inner surface of the lining along lines coinciding with the proposed crown and springline alignments. A galvanized steel pipe with a sharp cutting edge (O.D. = 152 mm, I.D. = 150 mm) connected to a hydraulic jack was chosen to perform the excavation

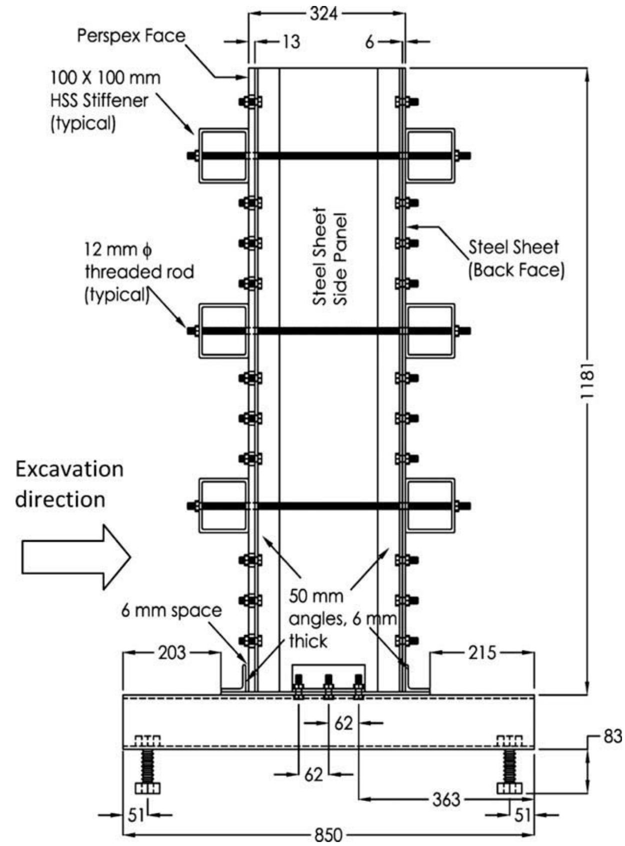

Fig. 2. Side view of the two-dimensional rigid box.

work within the model. A mix of bentonite clay and fine sand consolidated from slurry has been used to constitute the cohesive soil used throughout this study. To accelerate the consolidation process, a sand layer covered by geotextile sheet was used at the bottom of the box for drainage purposes. A summary of the lining and soil properties is provided in Table 1.

2.1. Test procedure

A schematic of the test setup and the tunnel excavation process is shown in Fig. 3. First, the shield casing was lubricated and ad-

Table 1
Lining and soil properties.

Lining properties	
Lining radius	0.073 m
Thickness	0.00025 m
Young's modulus[a]	65 GPa
Poisson's ratio[a]	0.3
Coarse sand layer	
Specific gravity (G_s)[a]	2.65
Effective size (D_{10})[a]	0.42
Uniformity coefficient (C_u)[a]	1.88
Friction angle[a]	35°
Dilation angle[a]	0°
Elastic modulus[b]	1.5×10^5 kN/m^2
Clay mix	
Moisture content (w_c)[a]	40%
Undrained shear strength (c_u)[a]	4 kPa
Saturated unit weight (γ_{sat})[a]	18 kN/m^3
Elastic modulus[a]	1150 kN/m^2

[a] Measured.
[b] Estimated using the available properties.

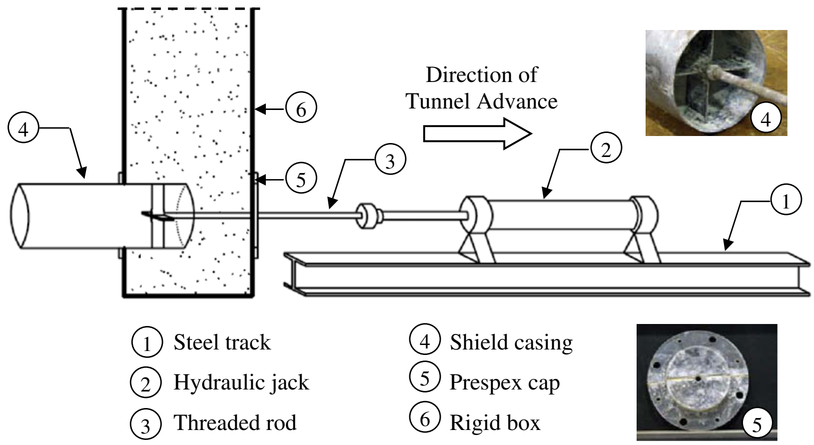

Fig. 3. Schematic of the test setup.

vanced by the hydraulic jack until the cutting edge was about 25 mm from the outer face of the cap covering the tunnel opening. At this time, the two halves comprising this cap were removed and the cutting edge of the casing was advanced into the tunnel opening. The casing was advanced at a rate of about 25 mm every 2 s. The cuttings inside of the casing were removed continually as the casing was advanced. When the cutting edge of the casing was approximately 6 mm from the inner surface of the rear cap, the nuts holding this cap in place were removed, along with the cap itself, and the casing was advanced until the leading edge had passed out of the hole by about 50 mm. This was to minimize the development of shear stresses on the excavated surface and any unnecessary increase in pressure against the inner face of the rear Perspex cap. The aluminum lining was then placed inside the casing, and the data acquisition system was armed. Finally, the casing was then advanced further, allowing the lining to become exposed to the walls of the soil cavity. At the instant that the surrounding soil came into contact with the lining, the data acquisition system was triggered, and recorded data for a period of 15 min. Pictures of the instrumented lining before and after installation are shown in Fig. 4.

2.2. Experimental results

A total of five tests were conducted, two control tests and three tests that involved overlying stiff layers. The test results are summarized below.

2.2.1. Control test

In order to investigate the effect of the overlying layer on the circumferential stresses developing in the tunnel lining, a control test was first conducted using a homogenous clay layer. Strain gauge readings are summarized in Fig. 5a. Averaged strains of 59 $\mu\varepsilon$ and -122 $\mu\varepsilon$ at the crown and springline, respectively, were measured. The bending stresses in the circumferential direction were calculated by multiplying the measured strains by the elastic modulus of the aluminum ($E_{\text{measured}} = 64$ GPa). Stresses at the crown and springline were found to be 4.9 MPa (tensile) and 7.8 MPa (compressive), respectively. Bending moments are then calculated based on the flexural theory ($M = \sigma.I/y$) using the converged strain values for all tests.

2.2.2. Coarse sand layer overlying the tunnel

A 150-mm thick layer (or one equivalent excavated diameter) of coarse sand (Quartz Industrial 2075) was introduced within the cohesive soil in order to investigate the effect of this layer on the response of the tunnel lining. A fully saturated geotextile fabric was used at the upper and lower interfaces between the soil and slurry in order to prevent against uncontrolled mixing between the strata. The layer was situated at elevations of 450 mm, 300 mm, and 150 mm above the tunnel crown, or three, two and one equivalent excavated diameters, respectively.

When the coarse sand layer was placed at a distance of 450 mm above the crown of the tunnel ($h/d = 3$), the average strain at the

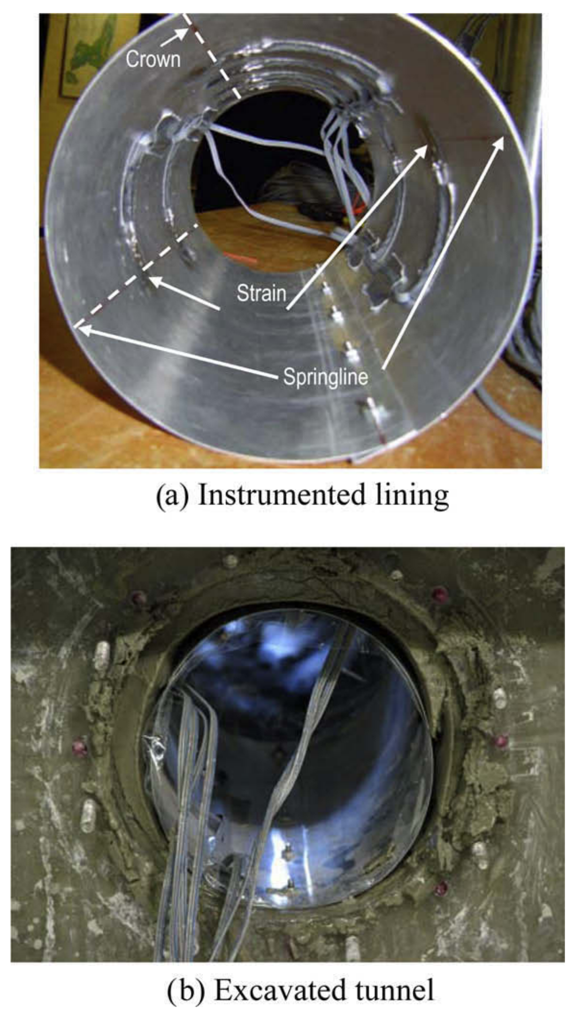

Fig. 4. Photographs of the aluminum lining before and after installation.

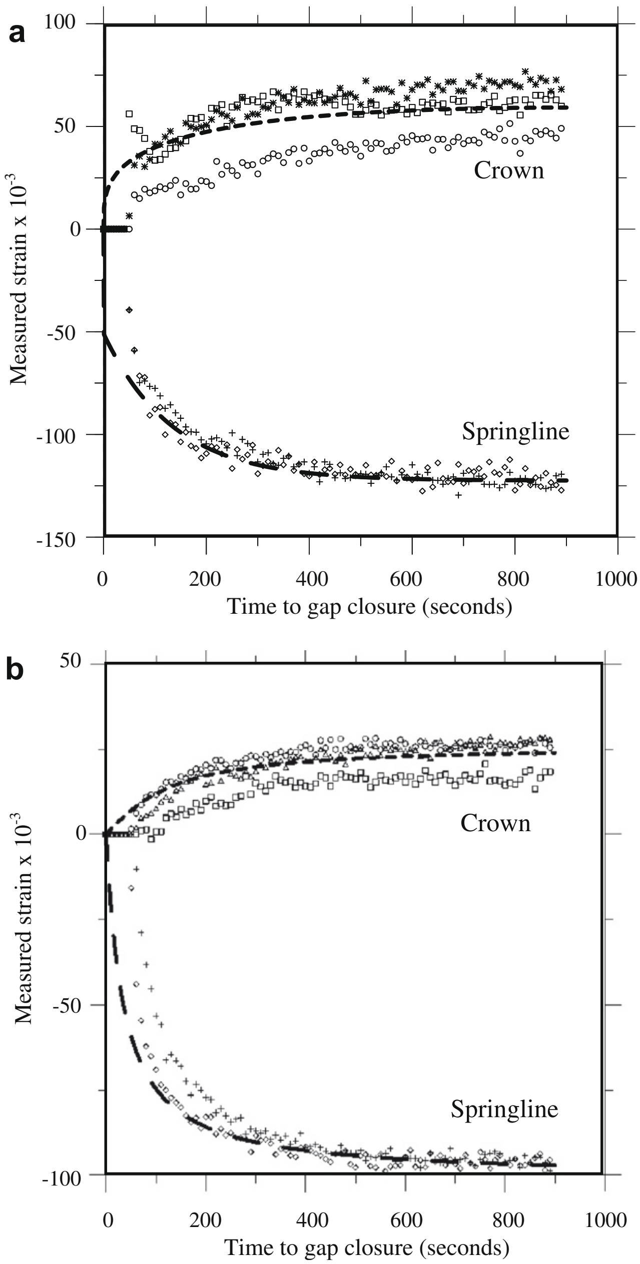

Fig. 5. Experimental results for: (a) the control test, (b) stiff layer located at $h/d = 3$.

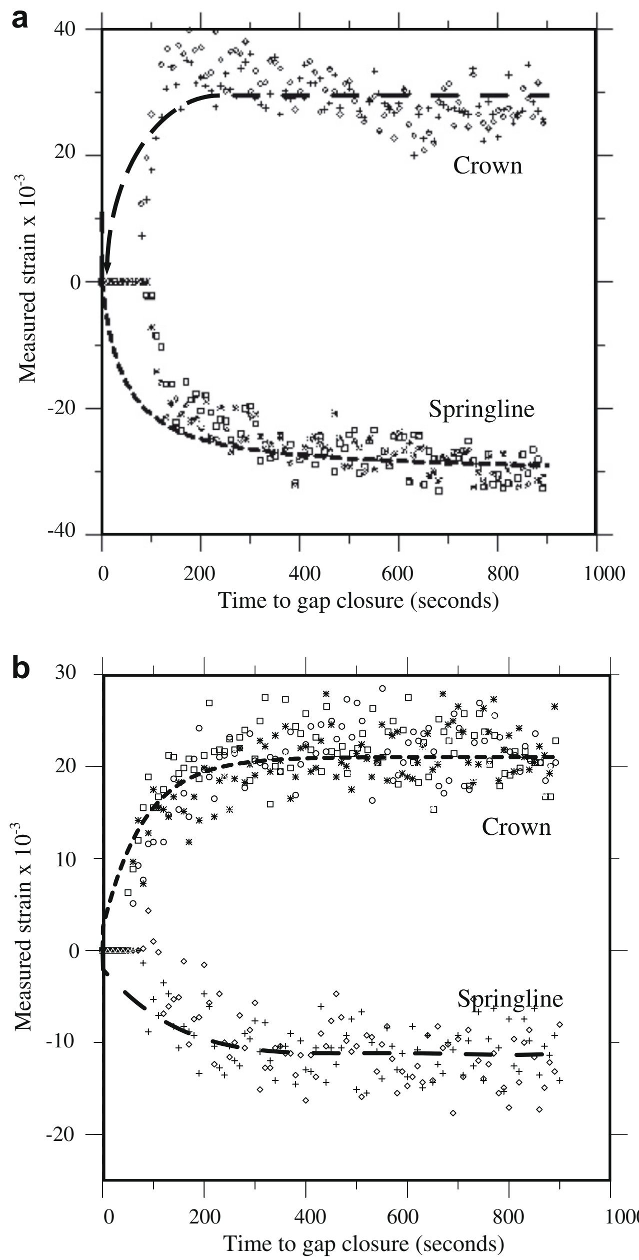

Fig. 6. Experimental results for stiff layer located at: (a) $h/d = 2$, (b) $h/d = 1$.

crown was found to be 32 $\mu\varepsilon$ and the average compressive strain at the springline was -89 $\mu\varepsilon$ as shown in Fig. 5b. This indicated a reduction in the lining stresses at the crown of about 20% and at the springline of about 30% as compared to the control test. As the sand layer is moved closer to the tunnel such that it is located 300 mm above the crown ($h/d = 2$) as shown in Fig. 6a, the average measured strains were found to be 30 $\mu\varepsilon$ and -29 $\mu\varepsilon$ at the crown and springline, respectively. The recorded strains in this case corresponded to a reduction of about 50% at the crown and about 75% at the springline as compared to the control test.

As the layer was further moved closer to the tunnel crown ($h/d = 1$), the strains at the crown and springline were significantly less than those read from the control test, registering at 21 $\mu\varepsilon$ and -11 $\mu\varepsilon$, respectively, as shown in Fig. 6b. These values were found to be significantly smaller as compared to the control test. The results were also of relatively smaller values as compared to the cases of $h/d = 3$ and 2 described above.

The results of the experimental investigation suggest that the introduction of a stiff soil layer over the tunnel can have a significant impact on the circumferential stresses in the lining.

3. Numerical analysis

A series of finite element analyses has been performed to examine the effect of the layer thickness, t, and location with respect to the tunnel crown, h, on the bending moment developing in the lining. The analyses were performed using Plaxis V8 software (Brinkgreve and Vermeer, 1998). The lining was modeled as linear elastic material whereas the soil was modeled using Mohr–Coulomb failure criterion with strength and stiffness parameters as listed in Table 1. It should be noted that the undrained shear strength of the clay material was measured using vane shear tests conducted at several locations around the tunnel before and after the experiment. The elastic modulus of the clay was back-calculated using the displacement measured around the tunnel during the tunnel excavation in the control test. The friction angle of the coarse sand was measured using direct shear tests. The model boundary conditions were selected such that they represent smooth rigid and rough rigid boundaries at the sides and base, respectively. 15-nodded triangular solid elements were used throughout the analysis. A typical finite element mesh is shown in Fig. 7.

The finite element model was validated by comparing the average lining stresses (and the corresponding bending moment) calculated based on the experimental data and the numerical predictions. The calculated and measured bending moments at the tunnel crown and springline are presented in Fig. 8. It can be seen that, although the numerical analysis overestimated the magnitude of the bending moment at some locations, it captured the deceasing trend of the moment as the stiff layer is introduced and moved towards the tunnel crown.

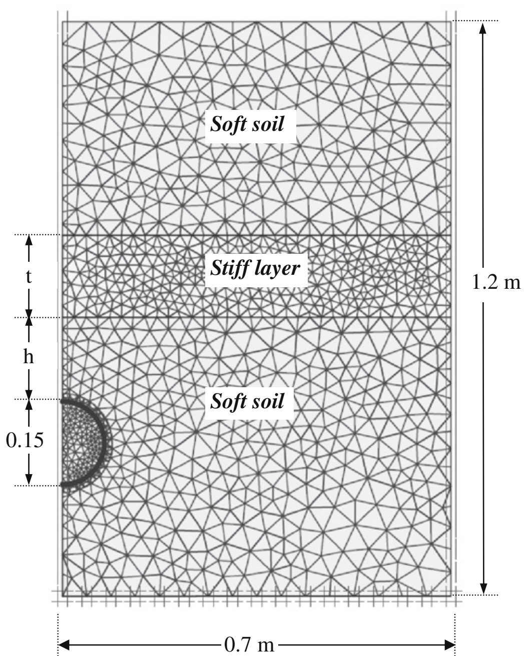

Fig. 7. Finite element mesh used in the parametric study.

4. Effects of stiffness ratio

The effect of the ratio between the stiffness of the upper layer and the saturated clayey soil on the tunnelling induced bending moment is numerically investigated and the results are presented in Fig. 9. The expression (G_{layer}/G_{soil}) has been used as a measure of the stiffness of the upper layer relative to that of the saturated clay. Values for the shear modulus G are determined based on the elastic modulus, E, assigned for both materials.

The thickness of the overlying layer has been kept constant, whereas the stiffness ratio has been increased in eight increments, namely, 25, 50, 75, 100, 150, 200, 250 and 300. This was achieved by increasing the shear modulus of the coarse sand layer. The results are presented for two different layer locations ($h/d = 1$ and 2) for comparison purposes.

It can be seen that the presence of a stiff layer above the tunnel can have a considerable impact on the bending stresses developing in the lining. Bending moments rapidly decreased when the stiffness of the overlying layer reached about 25 times that of the clay deposit. The changes in bending moment were less rapid when the relative stiffness increased from 25 to 100. Further increase in the relative stiffness was found to have insignificant effect on the bending moment. It was also observed that the location of the layer with respect to the tunnel lining can significantly affect the bending moments. Moving the stiff layer a distance of one tunnel diameter has resulted in additional reduction of the bending moment in the lining.

5. Effects of layer thickness

The relationship between the overlying layer thickness and bending moments developing in the lining is shown in Fig. 10. A stiffness ratio based on the soil properties as used in the experimental investigation was adopted in the analysis. Results indicated a rapid decrease in the magnitude of the bending moment when the thickness of the stiff layer increased to about one tunnel diameter. Additional increase in the layer thickness results was found to have insignificant impact on the calculated lining response. It was also found that the bending moment decreased more rapidly when the stiff was moved closer to the tunnel lining.

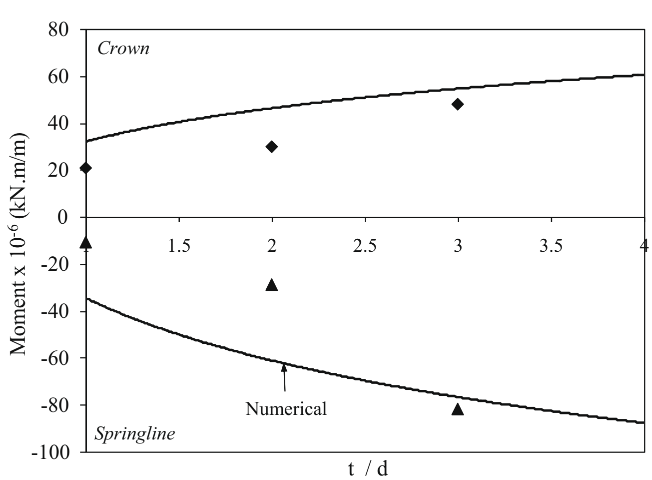

Fig. 8. Comparison between the calculated bending moments based on experimental results and FE analysis.

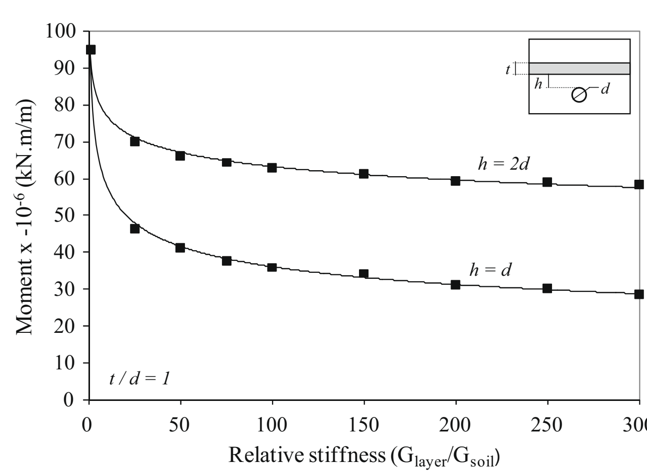

Fig. 9. Effect of the overlying layer stiffness on the bending moment at the springline.

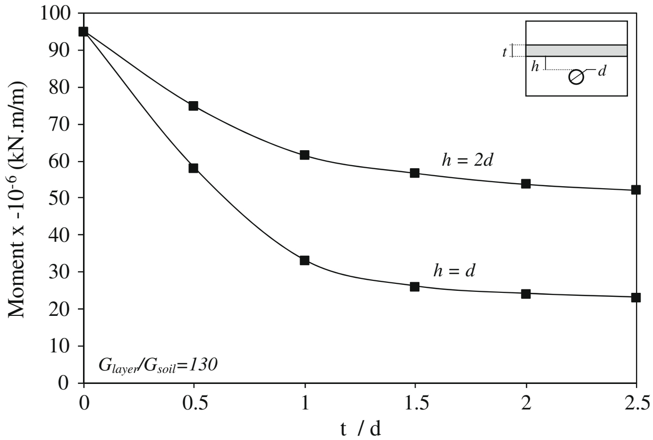

Fig. 10. Effect of the overlying layer thickness on the bending moment at the springline.

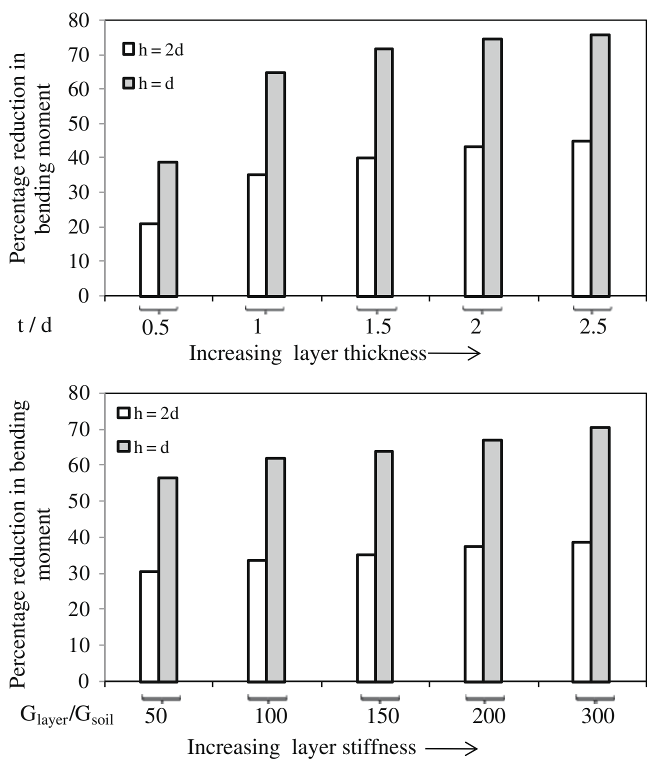

Fig. 11. Changes in bending moment at the springline due to the presence of a stiff layer above the tunnel.

Fig. 11 presents a summary of the percentage changes in the bending moment at the springline for the above investigated cases. For a given tunnel excavated in homogenous soil, a reduction in the bending moment of up to 50% due to the presence of a stiff layer at a distance 2*d* above the crown. This reduction in moment can reach up to 70% as the stiff layer is located at a distance 1*d* above the tunnel.

It should be emphasized that the above experimental and numerical results are based on simplified two-dimensional setup and only short term conditions have been examined. Further investigations are therefore needed to study the role of the overlying stiff layer on the three-dimensional behaviour near the tunnel face. In addition, the long term response and the final lining stresses require further evaluation.

6. Summary and conclusions

The structural behaviour of a tunnel liner installed in clayey soil and overlain by a course sand layer was investigated experimentally and numerically with the controlled parameters including stiffness ratios, layer thickness and location above the tunnel, and the following conclusions are drawn from the results with the conditions and assumptions given in this study.

(1) The investigations indicated that the response of a tunnel liner was generally affected by the presence of a stiff sand layer above the tunnel.
(2) The reduction in the bending moment was highly dependent on the relative stiffness (represented by the shear modulus ratio) between the sand layer and the clay deposit hosting the tunnel.
(3) Bending moment decreased more rapidly as the layer was closer to the tunnel with a maximum decrease of 70% from the case of homogenous clay when the stiff layer (of shear modulus ratio of more than 100) was located one tunnel diameter from the tunnel crown.

Further experimental and numerical investigations are needed to study other aspects of this interesting tunnelling problem including the cases when the stiff layer is crossing the tunnel section (mixed face conditions).

Acknowledgements

This research is supported by the Natural Sciences and Engineering Research Council of Canada (NSERC) under Grant Number 311971-06.

References

Addenbrooke, T.I., Potts, D.M., 2001. Finite element analysis of St. James Park greenfield reference site. In: Burland, J.B., Standing, J.R., Jardine, F.M. (Eds.), Building Response to Tunnelling, vol. 1. Thomas Telford, London, UK, pp. 177–194.
Attwell, P.B., 1978. Ground movements caused by tunnelling in soil. In: Proceedings of Large Ground Movements and Structures Conference, Cardiff. Pentish Press, London, pp. 812–948.
Bobet, A., 2001. Analytical solutions for shallow tunnels in saturated ground. Journal of Engineering Mechanics – ASCE 127 (12), 1258–1266.
Brinkgreve, R.B.J., Vermeer, P.A., 1998. PLAXIS-version 8, Finite-element code for soil and rock analyses. In: User's Manual, A.A. Balkema, Rotterdam, The Netherlands.
Chambon, P., Corte, J.F., 1994. Shallow tunnels in cohesionless soil: stability of tunnel face. Journal of Geotechnical Engineering 120 (7), 1148–1165.
Einstein, H.H., Schwartz, C.W., 1979. Simplified analysis for tunnel supports. Journal of Geotechnical Engineering - ASCE 105, 499–518.
Grant, R.J., Taylor, R.N., 2000. Stability of tunnels in clay with overlying layers of coarse grained soil. In: GeoEng 2000: An International Conference on Geotechnical and Geological Engineering, Melbourne, Australia.
Hagiwara, T., Grant, R.J., Calvello, M., Taylor, R.N., 1999. The effect of overlying strata on the distribution of ground movements induced by tunnelling in clay. Soils and Foundations 39 (3), 63–73.
Lee, K.M., Rowe, R.K., 1990. Finite element modeling of the three-dimensional ground movements due to tunnelling in soft cohesive soils: part 2-results. Computers and Geotechnics 10, 111–138.
Mair, R.J., Gunn, M.J., O'Reilly, M.P., 1981. Ground movements around shallow tunnels in soft clay. In: Proceedings of the Tenth ICSMFE, Stockholm, pp. 323–328.
Meguid, M.A., Saada, O., Nunes, M., Mattar, J., 2008. Physical modelling of shallow tunnels in soft ground: a review. Tunnelling and Underground Space Technology 23, 185–198.
Mitchell, J.K., 1976. Fundamentals of Soil Behaviour. John Wiley and Sons Inc., New York. p. 422.
Muir Wood, A.M., 1975. The circular tunnel in elastic ground. Geotechnique 1, 115–127.
Nomoto, T., Imamura, S., Hagiwara, T., Kusakabe, O., Fujii, N., 1999. Shield tunnel construction in centrifuge. Journal of Geotechnical and Geoenvironmental Engineering 125 (4), 289–300.

O'Reilly, M.P., New, B.M., 1982. Settlements above tunnels in the UK – their magnitude and prediction. In: Proceedings of Tunnelling' 82, IMM, London, pp. 173–181.
Peck, R.B., 1969, Deep excavations and tunneling in soft ground. In: Proceedings of the Seventh International Conference on Soil Mechanics and Foundation Engineering, Mexico City, pp. 225–290.
Quigley, R.M., Sethi, A.J., Boonsinsuk, P., Sheeran, D.E., Yong, R.N., 1985. Geologic control on soil composition and properties, Lake Ojibway clay plain, Matagami, Quebec. Canadian Geotechnical Journal 22 (4), 491–500.
Sagaseta, C., 1987. Analysis of undrained soil deformation due to ground loss. Geotechnique 37 (3), 301–320.
Schmidt, B., 1974. Prediction of settlements due to tunneling in soil: three case histories. In: Proceedings of the Second Rapid Excavation and Tunneling Conference, San Francisco, vol. 2, pp. 1179–1199.
Sharma, J.S., Bolton, M.D., Boyle, R.E., 2001. A new technique for simulation of tunnel excavation in a centrifuge. Geotechnical Testing Journal 24 (4), 343–349.
Swoboda, G., Mertz, W., Schmid, A., 1989. Three dimensional numerical models to simulate tunnel excavation. In: Proceedings of NUMLOG III. Elsevier Science Publishers Ltd., London, pp. 36–548.
Tanaka, T., Sakai, T., 1993. Progressive failure and scale effect of trapdoor problem with granular materials. Soils and Foundations 33 (1), 11–22.
Terzaghi, K., 1936. Stress distribution in dry and in saturated sand above a yielding trap-door. In: Proceedings of the International Conference on Soil Mechanics, Cambridge, MA, pp. 307–311.
Vardoulakis, I., Graf, B., Gudehus, G., 1981. Trap-door problem with dry sand: a statical approach based upon model test kinematics. International Journal for Numerical and Analytical Methods in Geomechanics 5, 57–78.
Venkatachalam, G., Naik, A.V., 1977. Interaction in tunnels in stratified media. In: Proceedings of the International Symposium on Soil Structure Interaction, University of Roorkee, India, pp. 33–38.
Verruijt, A., Booker, J.R., 1996. Surface settlements due to deformation of a tunnel in an elastic half plane. Geotechnique 46 (4), 753–756.